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Design of Steel Work For an Office Building

Civil Engineering

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DESIGN OF STEEL WORK FOR AN OFFICE BUILDING

BY

ALBERT PAUL STREFF

THESIS

For Degree of

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IN

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ALBERT PAUL STREFF

ENTITLED DESIGN OF STEELWORK FOR AN OFFICE BUILDING

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Civil Engineering

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DESIGN OF STEEL WORK FOR AN OFFICE BUILDING.

Art. 1. INTRODUCTION.

Statistics show that the fire losses, in the United States, are steadily increasing with the development and population of the country; and in view of this fact a new field has been recently opened in connection with building construction, namely that of fireproofing.

The earliest buildings of a fireproof nature in this country were probably those used for public purposes, and for these ample funds and large areas were allowed. They were usually of moderate height, and built of solid stone and brick arch construction making them fireproof. In this form of construction much valuable room is wasted by being taken up by thick walls and floors. A tall building of this nature would require very thick walls in the lower floors, especially in the first story and basement. This would not do in our large cities at the present time, where the ground area is limited in the business district, and where the floor space, especially in the first floor and in the basement is so valuable. It is the amount of floor space and the height above the ground level, upon which the rent of offices is based, the lower floors being more desirable than the upper ones. It was therefore this fact that created a demand for some other form of construction by which the area wasted in building the thick walls and floors could be available for other purposes.

Some material which could resist fire was of course needed and therefore steel was substituted for the stone and brick. By actual tests of fire it was found that steel alone would not

make a fireproof construction so a covering of terra cotta was added. Many forms and processes of manufacture are used in the making of this covering, but will not be discussed in this paper. The design of the steel skeleton will be the subject since it is that part of the construction that carries all the loads which come upon the building.

Art. 2. GENERAL DATA.

The building for which the steel work is to be designed is a five story and attic office building. The building is seventy-seven feet and nine inches long by forty-seven feet wide, and has a flat roof over most of it with the exception of a slight slope on three sides as shown in Fig.1. Plate I gives a general plan of the building.

In order to design the members used in the frame work it is necessary to determine the load which comes upon the building. These loads are divided into dead and live loads, or those that are always on the building and those that are liable to come on the building at any time. The dead load consists of the weights of the members themselves, the floor construction, partitions, masonry in the spandrels, trusses, roofing, glass and all stationary or permanent loads. The live loads are the moving loads or those that are not on the building at all times and are subject to change both in weight and location such as office furniture, safes, wind, and people.

Art. 3. DETERMINATION OF ROOF LOADS.

In figuring the loads it is necessary to begin at the top and work down since the loads of the roof and upper stories are

carried down, by the columns, to the footings. The dead load of the roof will be figured first, and the method of doing this is to find the weight per square foot of the truss, purlins, and covering, and multiply it by the square feet of the roof area. The roof is supported by trusses, since the attic floor is to have as little obstruction as possible, no columns being allowed to pass through the center. Since the truss cannot be designed until after the loads that come upon it are figured the exact weight cannot be obtained, but one must be assumed from other trusses of about the same span that has already been built. The covering is to be asphalt or a composition roof. The weight including trusses and purlins will be taken as forty pounds per square foot of roof surface. In order to take care of loads that might come upon the roof at any time, such as snow and wind, a live load of forty pounds per square foot is assumed on the roof joists, to provide for a maximum; and thirty and twenty-five on the girders, or trusses, and columns respectively.

Art. 4. DETERMINATION OF FLOOR LOADS.

The floor loads, like the roof loads, are made of dead and live loads. The dead load is figured by taking a section or bay, between four columns and calculating the actual weight. The two beams between the columns parallel to the length of the building will be taken as girders and the floor beams will frame into these as shown in Plates I, II, III and IV. The distance between floor beams is five feet and six inches, two beams being placed in the bay so that none will come at the center of the girder or be less than six feet center to center. The floor is of a ten

inch end construction tile arch, a side and an end view being shown in Fig. 2 and 3. The weight per square foot of this floor is determined as follows. The actual weight of the tile in the arch is thirty-three pounds per square foot. The steel will be assumed to consist of three nine-inch twenty-one pound I-beams and one fifteen-inch forty-two pound I-beam making a total weight of about seventeen hundred pounds. The area of the bay is two hundred and sixty-four square feet and dividing the total weight by this will give a weight of seven pounds per square foot for the steel.

The deadening to be used as shown in Fig. 2 and 3 is one inch of stone concrete and two inches of cinder concrete. Stone concrete weighs about 150 pounds per cubic foot and cinder concrete about 75 pounds per cubic foot, making a total of about twenty-four pounds per square foot. Seven eights of an inch maple flooring is used and weighs about four pounds per square foot. The plastering on the ceiling will be taken as weighing about five pounds per square foot of surface.

In an office building of this type the partitions are usually left out until rented or so arranged that they may be moved around and arranged by the occupants to suit their needs. This makes an additional dead load on the floor,and in this case it will be assumed to be seventeen pounds per square foot. This makes a total of ninety pounds per square foot as the weight of the floor.

The live loads on the floor are assumed and are taken large enough to provide for the maximum that might be expected to come on the floor any time. In most cities the building laws require

or specify a certain live load for each class of building. This load varies in different cities and in almost every case is assumed very much too large. The maximum load in the wholesale warehouse of Marshall Field and Co. in Chicago was found to be only 57 pounds per square foot on limited area. However, it is necessary to provide more than this in order to be on the safe side. The live loads per square foot on the floor beams of this building will therefore be assumed as follows: For the attic floor, fifty pounds, for the fifth to the second floor inclusive, seventy pounds, and for the first floor one hundred and twenty-five pounds.

The dead load on the girders, columns and footings will be the same as figured for the floor beams since the girders carry the floor-beams which in turn carry the floor. Also the columns and footings carry the floor-beams and girders and therefore carry the same dead load.

Art. 5. DETERMINATION OF THE COLUMN LOADS.

The live loads on the girders and columns are figured somewhat differently than those on the floor-beams. The girders carry the floor-beams and since they are fewer in number and farther apart they are not subjected to as much live load as are the floor beams. Just what percentage they carry compared to the floorbeams is hard to calculate. About eighty percent of the live load on the floor beams appears to be a good portion and one that has been used satisfactorly. Eighty percent will therefore be used for the design of the girders and this gives the following weights per square foot for the different floors:-

Forty pounds for the attic floor, fifty pounds for the fifth to the second floor inclusive, and one hundred pounds for the first floor.

About fifty percent of the live load on the floor beams is taken as the live load on the columns for the upper stories and is reduced about five percent for each floor down. The reason for this reduction is that the live loads on the upper floors are carried down and it is not probable that any one column will be loaded with a maximum live load at every floor. The live load for the attic story columns will be assumed as twenty five pounds per square foot, for the fifth story, thirty pounds, for the fourth story, forty five pounds, for the third story thirty five pounds, for the second story twenty five pounds, for the first story twenty pounds and for the basement story columns fifteen pounds per square foot.

Art. 6. DETERMINATION OF LOADS FOR FOOTINGS,SIDEWALKS AND STAIRS.

No live load will be figured for the footing, the dead load only being figured and this is taken as the dead load of the floors.

The dead load of the sidewalk per square foot is the actual weight of the concrete and steel. The sidewalk is six inches thick and one square foot of this thickness will weigh twenty-five pounds. The steel will weigh approximately ten pounds per square foot making a total weight of eighty-five pounds per square foot. The live load will be taken as three hundred pounds per square foot to provide for any possible load that may come upon it.

The live load on the stairs will be taken as one hundred pounds per square foot and the dead load as about twenty-five pounds per square foot. The dead load is the weight of the stringers, risers and treads. The marble weighs about fifteen pounds per square foot and the steel about ten pounds per square foot making a total of twenty-five pounds.

Table I gives all these dead and live loads that come upon the beams, girders, columns and footings for all the floors including the roof. It also gives the live and dead loads for the sidewalk and stairs. A good idea of the amount of load and the distribution of the same can be readily seen by turning to this table.

Art. 7. ELEVATOR LOADS.

The load of the elevator on the columns is found by doubling the weight of the moving parts of the elevator, such as car-load and counter weight, and adding fifty percent to the weight of machinery and hangings to cover impact. The distribution of the load is shown in Fig. 4. This method of figuring the loads is used by the Otis Elevator Company for elevators with the driving machinery on the roof.

Art. 8. SPANDREL LOADS.

The weight or load on the spandrel girders will be the dead load weight of the spandrel. This includes the weight of the masonry, terra cotta, the glass and sash, and steel. One of the spandrel sections will be figured in order to show the method of proceedure. Refering to Fig. 10 a better idea of the fifth floor spandrel section can be had. The cross section area of this

spandrel is eighteen square feet, being eleven feet in height and approximately fifteen inches thick. This includes that part of the terra cotta which projects beyond the front wall. There are three windows in the spandrel the glass of which is 26 by 60 inches. Adding eight inches to the width and eleven to the length gives a total brick-opening of about fifty square feet. The cross section of the columns is assumed to be about six square feet and are eleven feet long.

The weight of the masonry and terra cotta in the spandrel is estimated as follows: terra cotta and brick backing 112 pounds per cubic foot, glass and metal sash 5 pounds per square foot of surface. There are 18 x 10.75= 198 cubic feet of brick and terra cotta, and from this is subtracted 50 x 1.25 = 62.5 cubic feet for windows. The amount to be added for the columns is 11 x 6 = 66 cubic feet. The total munber of cubic feet of brick and terra cotta in the section is 198 + 66 - 62.5 = 202. The weight of the terra cotta and brick is 202 x 112 = 22,650 pounds; the weight of the glass and sash is 50 x 5 = 250 pounds, making a total weight of 22,900 pounds.

Art. 9. DESIGN OF THE ROOF TRUSSES.

After the loads that come on the roof, floor beams, girders, columns and footings are figured the next step is to design these members. The same outline will be followed as was used in determining the unit loads. The roof will be designed first, then the floor beams, girders, columns, spandrels, and finally the footings.

The stress diagram of the roof truss "A" is shown in Fig. 5. The member X - 9 of this truss will now be designed. Since the

member is in compression the allowable unit stress will be somewhat reduced. It is based on the formula:

$$S = 16,000 - 70 \times l/r$$

in which

l = Length of the member in inches;

r = Least radius of gyration.

The least radius of gyration shall not be less than 1/125. In this case "l" is 11 x 12 = 132 inches and the least radius of gyration is 132/125 = 1.06

Two angles 4 x 3 x 7/16 placed back to back will be tried. The long legs will be placed together with a space of 3/8 inches between them. The area of the two angles is 5.74 square inches and the least radius of gyration is 1.26. The allowable unit stress as figured from the formula is 16,000 - 70 x 132/126 = 8,660 pounds. The stress is 44,900 pounds and the required area is 44,900/8,660 = 5.17 square inches. As this is very near the area of the two angles assumed, they will be used for this member.

Member A - 9 is a tension member and has a stress of 31,700 pounds. The allowable stress is 16,000 pounds per square inch. The required area is 31,700/16,000 = 2.0 square inches. The gross area of a 4 x 4 x 3/8 angle is 2.86 square inches and from this is subtracted the area of one unit hole. The net area is 2.86 - (3/4 + 1/8) x 3/8 = 2.53 square inches. This is greater than the required area, and these angles will therefore be used.

The other members of the trusses are designed in a similar manner. The stresses in and the sizes of the members are given

in Table II.

The design of the purlins is as follows: the distance center to center of trusses is 14 feet 9 inches, and the panel length is 7 feet 6 inches, making an area of 14.75 x 7.5 = 110.6 square feet. From Table I the load is 80 pounds per square foot. The total panel load is 110.6 x 80 = 8,850 pounds or 4.43 tons. From the Carnegie handbook page 71 an 8-inch,18-pound I-beam will carry a uniform load of 5.06 tons safely for a span of 15 feet, and since a 7-inch, 20-pound I-beam will not carry the load of 4.43 tons this will be used. The other roof joists are designed in the same way. The sizes are given in Fig. 6.

Art. 10. DESIGN OF THE FLOOR BEAMS.

There are several methods of designing the floor beams. These are by taking either the safe load on the beam, the maximum moment, or the section modulus method. This last method will be used in the design of the floor beams, and those in the bay between columns 5,6,7 and 8 for the fifth floor will be the ones designed.

The formula used in the design of the floor beams is:

$$S = M/f$$

in which

M = The bending moment in inch-pounds;

S = Section modulus in inch units;

f = Allowable unit stress per square inch in extreme fibres, usually taken at 16,000 pounds per square inch.

But $M = Wl/8$ for a beam supported at both ends and uniformly

loaded, "W" being the total load in pounds. Hence

$$S = Wl \times 12/8f$$
$$= 3Wl/2f$$

Also, as $W = dwl$, the formula becomes:

$$S = 3dwl^2/2f$$

in which

d = Distance center to center of beams, in feet;

w = Load per square foot in pounds.

Refering to Table I it will be seen that "w" the load per square foot in pounds for the fifth floor is 160, and the distance center to center of beams is 5.5 feet. Here $l = 16$ feet and substituting in the above formula

$$S = 3 \times 5.5 \times 160 \times 256/ 2 \times 16,000$$
$$= 21.15$$

From the Carnegie hand book, page 98, the section modulus nearest this is 22.6 which is for a 9-inch 30-pound I-beam. Since this is nearest to the value 21.15 this beam will be used.

The floor beams for the other bays and the other floors are designed in the same way, the only difference being in the load "w" and distance center to center of beams. For the size of beams see floor framing plans, Plates I to V.

Art. 11. DESIGN OF THE FLOOR GIRDERS.

The floor girders will be designed by the section modulus method in a similar manner as the floor beams. The loading on these girders is somewhat different from that of the floor beams, in that the load from the floor beams is concentrated at two points

The live load on the girders for the same bay, as taken

above for the floor beams, is taken from Table I, and is 140 pounds per square foot. The concentrated load on the girder at the third points is, see Fig. 7, 5.5 x 15.5 x 140 = 1195 pounds, say 12000pounds. The reaction is:

$$\begin{aligned} R &= 12000 \times 11.0 - 12000 \times 5.5/16.5 \\ &= 12000. \end{aligned}$$

The moment at the center of the beam is 12000 x 8.25 - 12000 x 2.75 = 66000 pound feet, and the moment under one load is 12000 x 5.5 = 66000 pound feet. The moment in pound inches is 66000 x 12 = 792000. The section modulus "S" is 792000/16000 = 49.5. Page 98, Carnegie handbook, gives the section modulus of a 15-inch 42-pound I-beam as 58.9, and since this is the nearest to 49.5 this beam will be used. The remaining girders are figured in a similar manner, and are given on floor framing plans. Table I and the floor plans will give the loading and lengths.

Art. 12. DESIGN OF COLUMNS.

There are several forms of steel columns used in building construction, the more prominent forms now used being channels connected by plates or lattice, plate and angles in various forms, and Z-bar columns. For light loads and a building of this height channel columns are the most common sections employed. Channels placed back to back and connected by plates will be used for the columns for this building.

The loads on the column are those of the roof, spandrels, floor, column casing, weight of columns above, and elevator and tank loads. The live load on the column is taken as about

fifty per cent of the load on the floor as will be seen in Table I.

Probably the best way to describe the method of designing a column is to take one and carry out the computation. Column No. 2 will be taken as an example since it is a typical one. The columns will be taken as extending through two stories, since this eliminates the connection at every other floor and tends to make the column more rigid. The size of the column should be so proportioned that the least radius of gyration is greater than the length in inches divided by one hundred and twenty five.

The Carnegie Steel Company's handbook, pages 137 to 142, gives the safe loads in tons of 2000 pounds for these channel columns with square ends. A factor of safety of four is used, and the allowable unit stress is figured from the formula $S = 17,000 - 57 \times l/r$ for lengths over 90 radii and less than 125 radii.

The columns are spliced at the second and fourth floors. These splices will be placed about one foot above the floor line. This will make the length of column No 2 as follows: the first and basement story section 32 feet, the second and third story section 22 feet and the fourth and fifth story section 25 feet.

The loads in the fourth and fifth story column No. 2 are: roof load of 22,200 pounds, weight of attic floor spandrel = 12,200 pounds, attic floor load 114 x 100 = 11,400 pounds, fifth floor spandrel 22,800 pounds and weight of column estimated to to be 400 pounds, making a total weight of 85,400 pounds or 42.7 tons.

The length of this column is twenty five feet and therefore

the least radius of gyration cannot be less than 25 x 12 / 125 = 2.4 . On page 138 of the Carnegie handbook it will be seen that two 7-inch 9.75-pound channels connected by two 1/4 inch plates have a least radius of gyration of 2.67 and will carry safely a load of 55.9 tons for a length of 26 feet. As this is the lightest section that will carry the load safely it will be ueed.

The second and third story sections of this same column is designed in a similar manner. The spandrel loads, floor loads, weight of columns and the load of the column alone are used in designing this part of the column. There are: load from column alone = 85,400 pounds, third and fourth floor loads = 27,400 pounds and weight of third and fourth floor spandrels = 43,300 pounds, making a total weight of 157,100 pounds or 78.55 tons.

The length is twenty-two feet and the least radius of gyration is 22 x 12 / 125 = 2.12. Two 8-inch 11.25-pound channels connected by two 3/8 inch plates have a least radius of gyration of 3.01 and carries safely a load of 85.2 tons. This section will be used.

The bottom length of the column is designed in this same manner. The total load on it is 227,500 pounds or 113.75 tons. Two 10-inch 15-pound channels connected by two 1/2 inch plates will carry the load safely and will therefore be used.

The remaining columns are designed in a similar manner. The loads and sections used are found on the column sheets.

Art. 13. DESIGN OF FOOTINGS.

The total dead load of the roof, floors, spandrels and columns is carried to the footing. No live load is considered in the design as is seen in Table I.

The total load on the footing for column No. 2 is taken from the column sheet and is 19,820 pounds or 99.1 tons. The allowable bearing on concrete is from 5 to 15 tons per square foot, so an average of 10 tons will be used. The bearing area on the concrete for this column is 99.1/10 = 9.91 square feet. This makes the base 3 feet 2 inches square. A plate one inch thick and 3 feet 2 inches square will be used for the base.

Under the column at the west end of the building it is necessary to put cantilever beam footings in order to keep the footings of columns 6, 7, 18 and 19 inside the building line. The footing under columns 7 and 8 will be figured as an example. Fig. 8 shows this footing.

The load of column No. 8 is 67.6 tons and of column No. 7 is 111.35 tons. By taking moments about R_2 the reaction R_1 is 67.6 x 13.5 - 111.35 x 30 / 13.5 = 42.3 tons. R_2 is found by taking moments about R_1 and is 111.35 x 16.5 /13.5 = 136.0 tons.

The maximum bending moment in the beam occurs at R_2 and is (67.6 x 13.5 - 42.3 x 13.5) x 12 x 2000 = 8,000,000 inch-pounds. The section modulus is 8,000,000 / 16,000 = 500. Since this is larger than the section modulus for the largest standard I-beam, more than one must be used. Three 24-inch 80-pound I-beams have a section modulus of 532 and will therefore be used.

The sidewalk beams will be figured in a similar manner to the floor beams. The formulas $S = 3dwl^2/2f$ of page 11 will be used. The section modulus S is 3 x 5.5 x 324 x 385 / 32,000 = 64.4. A 15-inch 60-pound I-beam has a section modulus of 81.2, and will be used. The sidewalk beams are all 15-inch 60-pound I-beams.

Art. 14. DESIGN OF SPANDRELS.

The spandrels will be designed by the safe uniform load method. The attic floor spandrel will now be designed. The weight of the material of the spandrel is uniformly distributed and is 12,200 pounds. The uniform load of the floor is 9,600 pounds making a total of 21,800 pounds or 10.9 tons. From Carnegie it is seen that two 12-inch 20.5-pound channels will carry this load.

The remaining spandrels are designed in a similar manner, and are found in Figs. 9, 10, 12, 14, 15 and 16.

Art. 15. CONLUSION.

In conclusion it may be said that the method used in the design of the steel work of this building is only one of several. The live loads on the floors is taken as a fair average of those specified by several of the large cities. In designing the steel work for an office building for a certain locality the loads specified for that place must be used. The specifications used in this design are those compiled by The American Society of Civil Engineers.

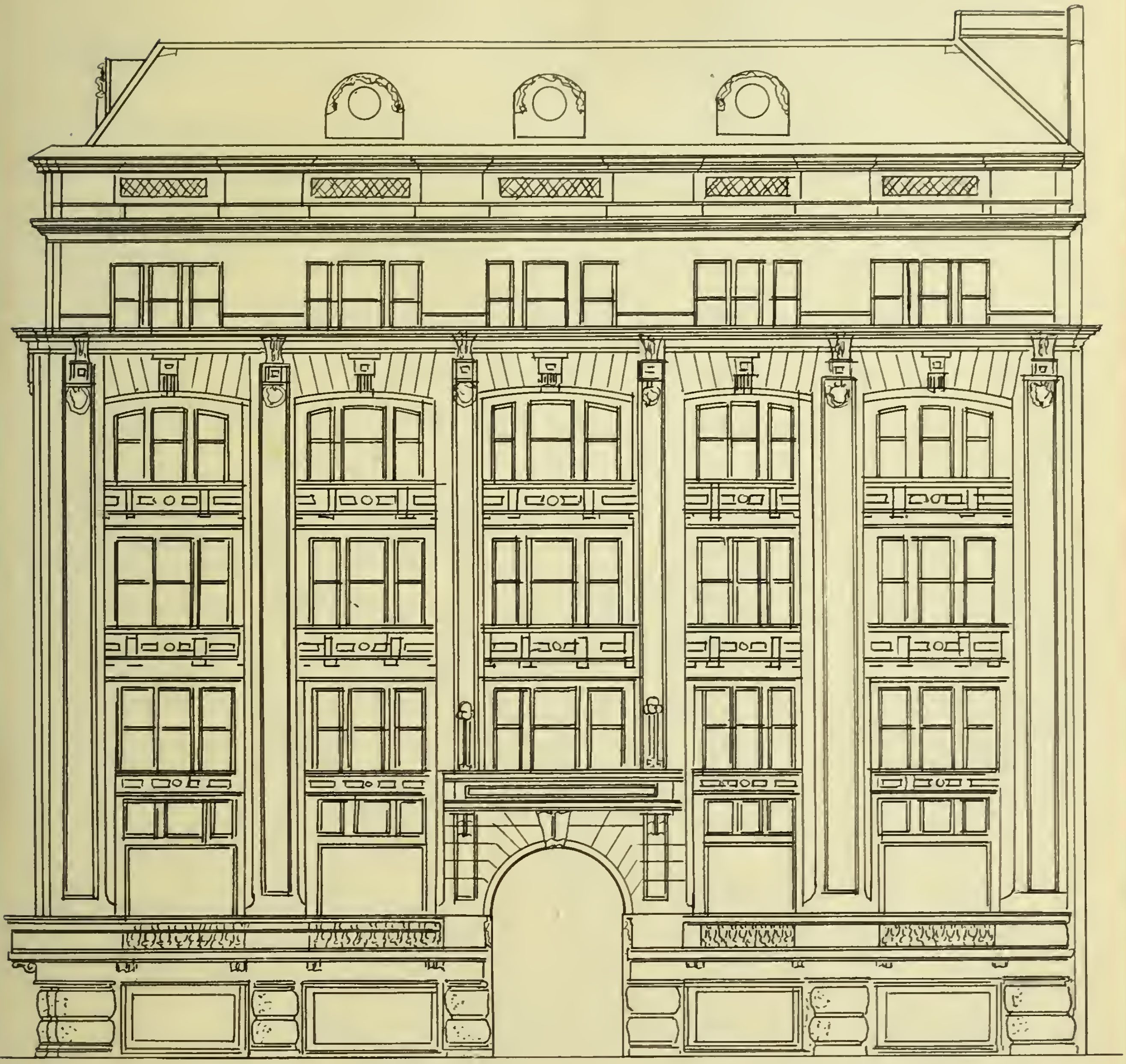


FIG I.

FRONT ELEVATION

TABLE I

LOADS.

	LOAD	BEAMS Lbs. per Sq. Ft.	GIRDERS Lbs. per Sq. Ft.	COLUMNS Lbs. per Sq. Ft.	FOOTING Lbs per Sq. Ft.
Roof	Dead	40	40	40	40
	Live	40	30	25	
Total		80	70	65	40
Attic	Dead	70	70	70	70
	Live	50	40	30	
Total		120	110	165	110
5th. Floor	Dead	90	90	90	90
	Live	70	50	45	
Total		160	140	305	200
4th. Floor	Dead	90	90	90	90
	Live	70	50	35	
Total		160	140	430	290
3rd Floor	Dead	90	90	90	90
	Live	70	50	25	
Total		160	140	545	380
2nd Floor	Dead	90	90	90	90
	Live	70	50	20	
Total		160	140	655	470
1st Floor	Dead	90	90	90	90
	Live	125	100	15	
Total		215	190	760	560
Sidewalk	Dead	85	85	85	85
	Live	300	250	175	
Total		385	335	260	85
Stairs	Dead	25	25	25	25
	Live	100	80	60	
Total		125	105	85	25

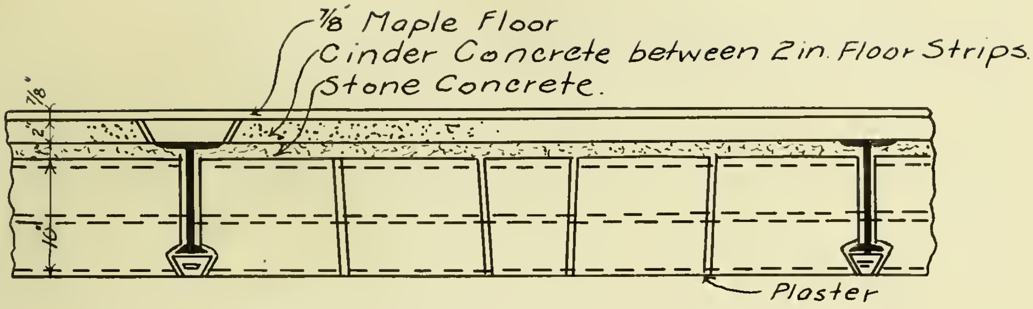


Fig. 2. Side view of Section through Floor Arch.

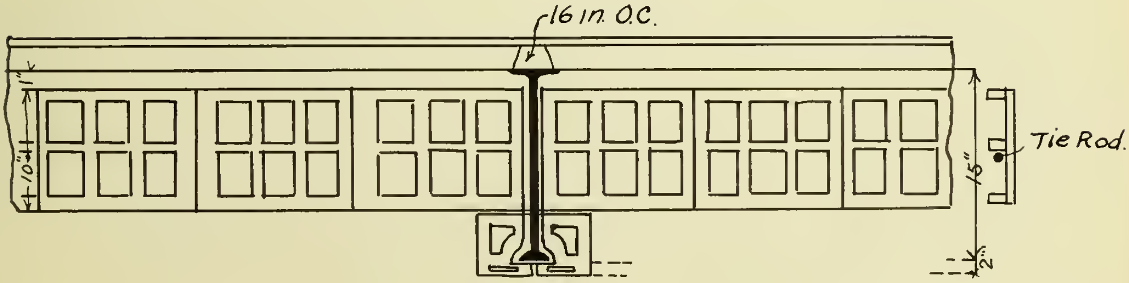
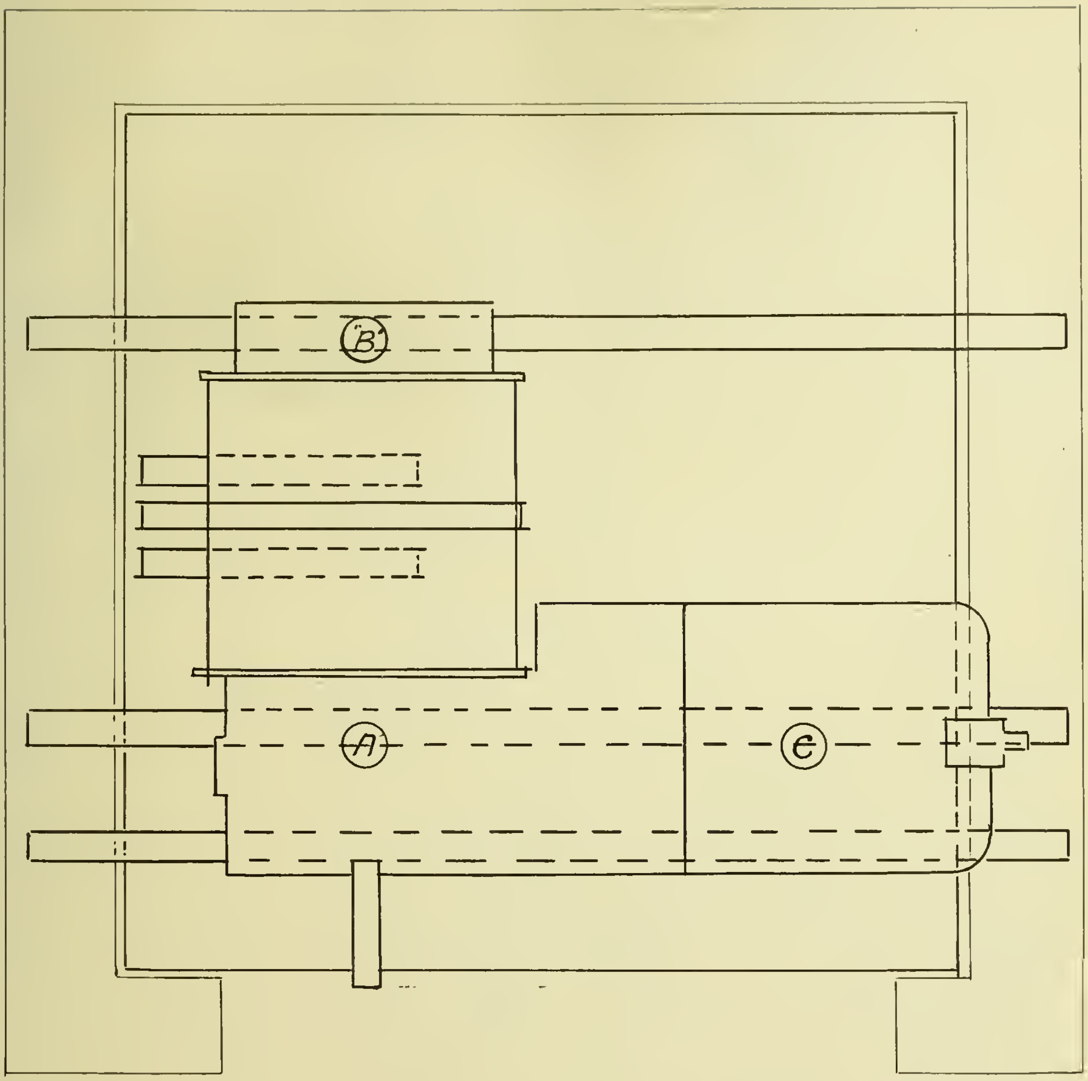
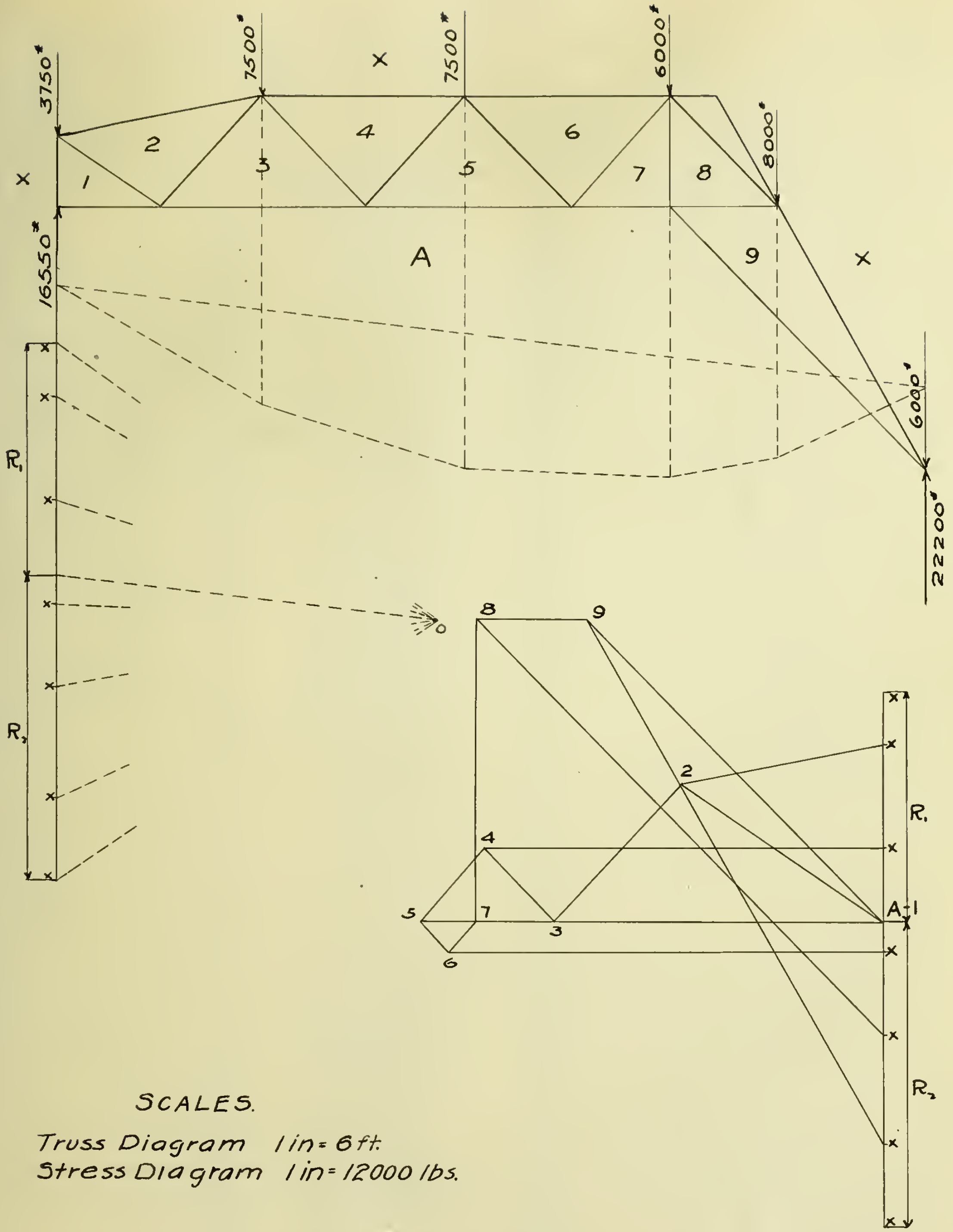


Fig. 3. End View of Section.



Elevator Loads Distributed at Points "A" "B" + "C"

Fig 4. Showing Distribution of Elevator Loads.



SCALES.

Truss Diagram 1 in = 6 ft.
Stress Diagram 1 in = 12000 lbs.

Dead Load 40 lbs. per sq. ft.
Live Load 30 lbs per sq. ft.
Distance c. to. c. Columns 31.5 ft.
Distance c. to. c. Trusses 14.25 ft.

Fig. 5 Stress Diagram Roof Truss "A"

TABLE II.

Stresses and Section of Roof Truss Members.

TRUSS	MEMBER	STRESS	LENGTH INCHES	SECTION
A	X-1	-16550	30	2 L^s 3×2½×¼
	1-2	+17700	54	1 L 3×3×5/16"
	X-2	-14900	90	2 L^s 3×2½×¼
	2-3	-13500	66	2 L^s 3×2½×¼
	A-3	+23800	90	1 L 3×3×3/8"
	3-4	+ 7100	66	1 L 2½×2½×¼
	X-4	- 28600	90	2 L^s 3½×3×5/16
	A-5	- 7100	66	2 L^s 2½×2×¼
	A-5	+33600	90	1 L 3½×3½×3/8
	5-6	- 3100	66	2 L^s 2½×2×¼
	X-6	-31500	90	2 L^s 3½×3×3/8
	6-7	+ 3100	66	1 L 2½×2½×¼
	A-7	+29400	45	1 L 3½×3½×3/8
	7-8	+23000	48	1 L 3×3×5/16
	X-8	- 42800	66	2 L^s 4×3×5/16
	8-9	+ 7500	45	1 L 2½×2½×¼
	X-9	-44900	132	2 L^s 4×3×7/16
	A-9	+31700	157	1 L 4×4×3/8
B	X-1	- 21500	36	2 L^s 3×2½×¼
	1-2	+ 23600	57	1 L 3×3×5/16"
	X-2	- 18200	88	2 L^s 3×2½×¼
	2-3	- 20800	65	2 L^s 3×2½×¼
	A-3	+ 32100	85	1 L 3½×3½×3/8
	3-4	+ 13700	65	1 L 3×3×¼
	X-4	- 41200	85	2 L^s 4×3×5/16
	4-5	- 13700	65	2 L^s 3×2½×¼
	A-5	+ 50500	85	2 L^s 3½×3½×5/16
	5-6	- 7700	65	2 L^s 2½×2×¼
	X-6	- 45300	85	2 L^s 4×3×3/8
	6-7	+ 7700	65	1 L 2½×2½×¼
	A-7	+ 40000	43	2 L^s 3½×3½×5/16"
	7-8	+ 33800	48	1 L 4×4×5/16
	X-8	- 60600	65	2 L^s 5×3½×7/16
	8-9	+ 7000	43	1 L 2½×2½×¼
	X-9	- 63200	133	2 L^s 5×3½×7/16
	A-9	+ 47300	160	1 L 4×4×½"
C	X-1	- 9600	90	2 L^s 2½×2×¼"
	1-2	+ 800	66	1 L 2½×2½×¼"
	X-2	- 5400	90	2 L 2½×2×¼
	2-3	+ 8000	66	1 L 2½×2½×¼"
	A-3	+ 10800	45	1 L 2½×2½×5/16"
	3-4	+ 11500	48	1 L 2½×2½×5/16
	X-4	- 22500	66	2 L^s 3×2½×5/16
	4-5	0	45	1 L 2×2×¼
	X-5	- 22500	132	2 L^s 3×2½×5/16
	A-5	+ 15900	157	1 L 3×3×5/16

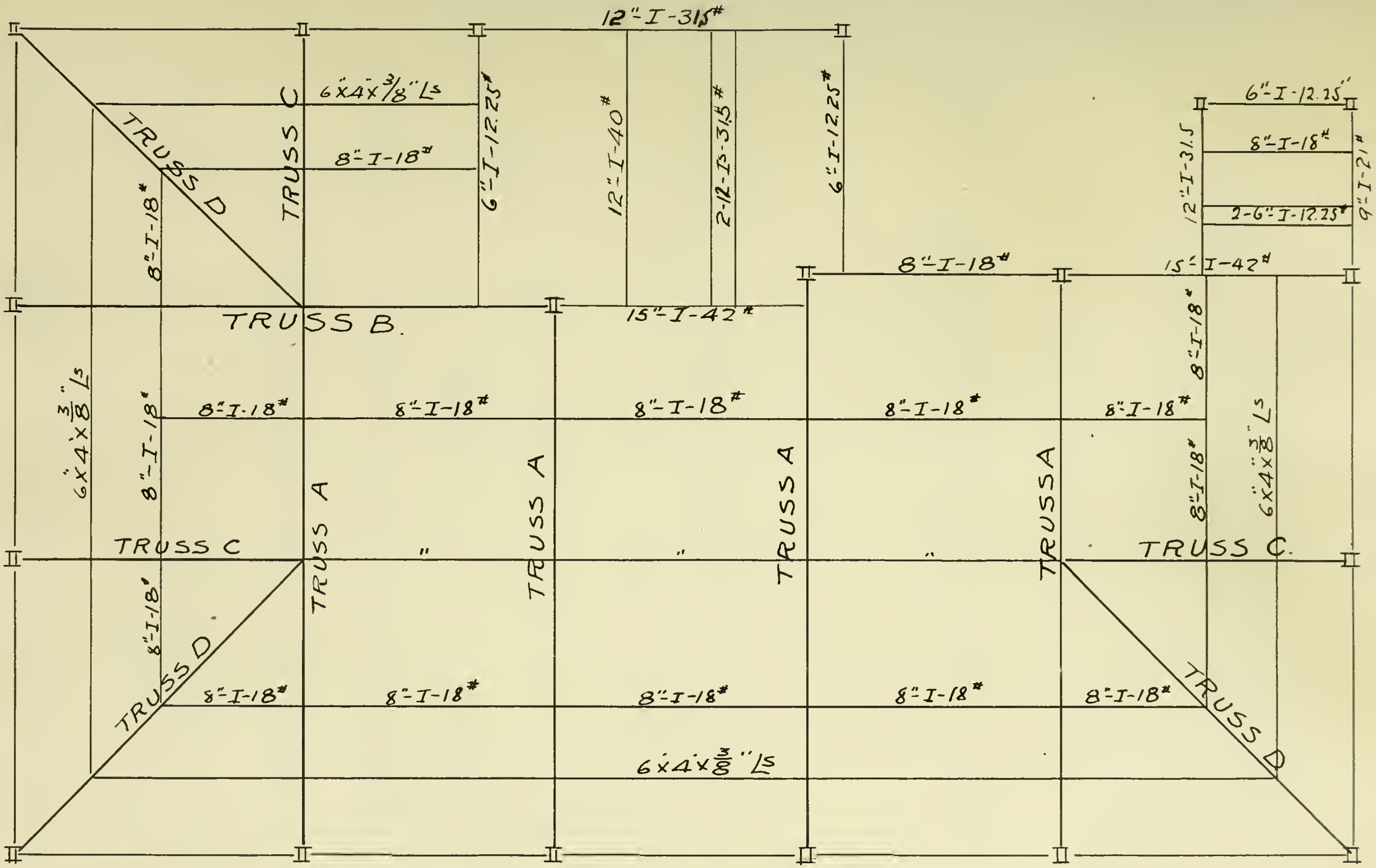


Fig. 6. Roof Framing Plan.

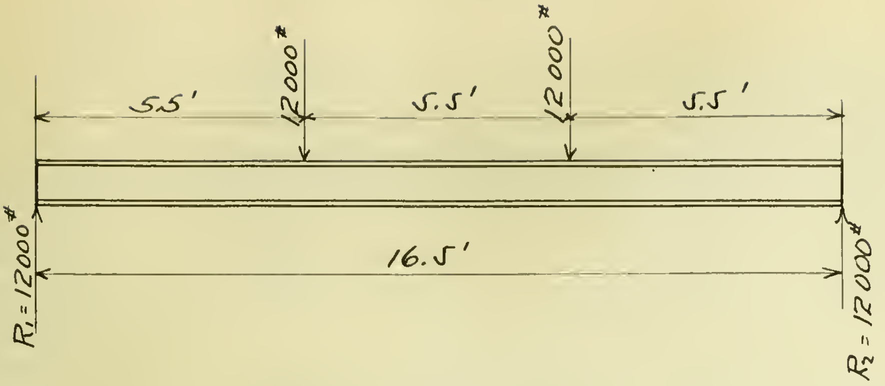


Fig. 7. Showing Concentrated Loads of Floor Beams on the Floor Girders.

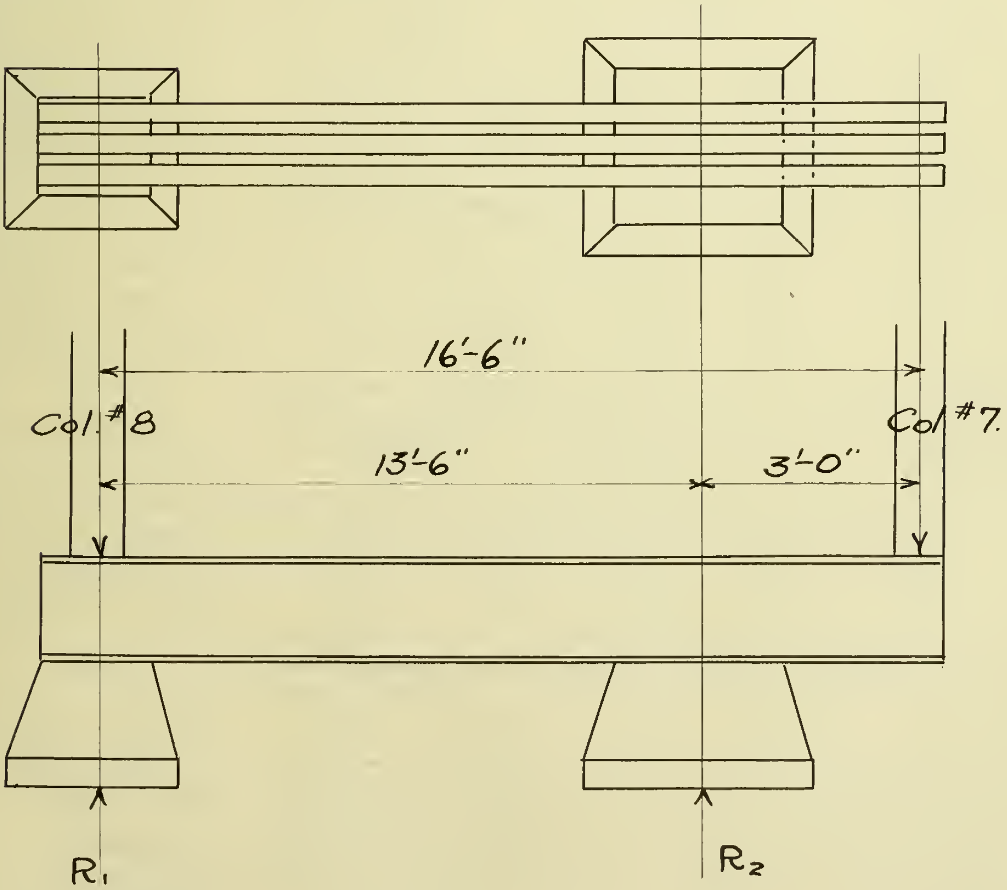


Fig. 8 Cantilever Column Footing.

COLUMN SHEET.

		Column No 1.		Cols. No. 2-3-4-5.		Column No 6.	
		Load on Column.	Load on Footing	Load on Column	Load on Footing	Load on Column	Load on Footing
5TH. FLOOR	Roof Load	4300	2600	22200	12700	4300	2600
	Spandrel	20200	20200	12200	12200	48100	48100
	Floor Load	6500	4600	11400	8000	6500	4600
	Elevators						
	Weight of Col.	300	300	400	400	400	400
	Total	31300	27700	46200	33300	59300	55700
4TH. FLOOR	Floor Load	9100	5900	16000	10300	9100	5900
	Spandrel	32400	32400	22800	22800	27000	27000
	Weight of Col	300	300	400	400	400	400
	Total	73100	66300	85400	66800	95800	89000
	Material of Column.	2-8"-11.25#-[s Latticed		2-7"-9.75# [s 2-¼" Pls.		2-8"-13.75#-[Latticed.	
3RD. FLOOR	Floor Load	8200	5900	14300	10300	8200	5900
	Spandrel	34300	34300	23900	23900	31200	31200
	Weight of Col.	500	500	500	500	500	500
	Total	116000	107000	124100	101500	135700	126600
2ND. FLOOR	Floor Load	7500	5900	13100	10300	7500	5900
	Spandrel	32600	32600	19400	19400	28700	28700
	Weight of Col.	500	500	500	500	500	500
	Total	156700	146000	157100	131700	172400	161700
	Material of Column.	2-8"-11.25# [s 5/16" Pls.		2-8"-11.25# [s 2-5/16" Pls.		2-8"-11.25# [s 2-3/8" Pls.	
1ST FLOOR	Floor Load	7200	5900	12500	10300	7200	5900
	Spandrel	32600	32600	19400	19400	28700	28700
	Weight of Col.	1200	1200	1100	1100	1200	1200
	Total	197700	185700	190100	162500	209500	197500
BSMT. FLOOR	Floor Load	6800	5900	12000	10300	6800	5900
	Spandrel	44000	44000	24300	24300	35800	35800
	Weight of Col.	1200	1200	1100	1100	1200	1200
	Total	249700	236800	227500	198200	253300	240400
	Material of Column	2-10"-15#-[s 2-9/16" Pls.		2-10"-15#-[s 2-½" Pls.		2-10"-15# [s 2-9/16" Pls.	
FOOTING	Footing	236800		198200		240400	

COLUMN SHEET.

		Col. No 7.		Cols. No. 8-9-10-11-14		Col. No 12.	
		Load on Column.	Load on Footing	Load on Column	Load on Footing	Load on Column	Load on Footing
5TH. FLOOR	Roof Load	8400	4800			7800	4800
	Spandrel	17400	17400			49000	49000
	Floor Load	14500	10000	24200	17000	12000	8400
	Elevators						
	Weight of Col.	300	300	1300	1300	500	500
	Total	40600	32500	26500	18300	69300	62700
4TH FLOOR	Floor Load	18600	12400	33800	21800	16800	10800
	Spandrel	24000	24000			23000	23000
	Weight of Col.	300	300	1300	1300	500	500
	Total	83500	69200	61600	41400	109600	97000
	Material of Column.	2-8"-11.25# [s 2-¼" Pls.		2-7"-9.75# [s 2-¼" Pls.		2-8"-11.25# [s 2-¼" Pls.	
3RD. FLOOR	Floor Load	16400	11800	30300	21800	15000	10800
	Spandrel	28200	28200			24000	24000
	Weight of Col	500	500	1400	1400	500	500
	Total	128600	109700	93300	64600	149100	132300
2ND FLOOR	Floor Load	15000	11700	27800	21800	13800	10800
	Spandrel	24900	24900			19400	19400
	Weight of Col.	500	500	1400	1400	500	500
	Total	169000	146800	122500	87800	182800	163000
	Material of Column	2-8"-11.25# [s 2-3/8" Pls.		2-7" 9.75# [s 2-1/4" Pls.		2-8"-11.25# [s 2-7/16" Pls.	
1ST FLOOR	Floor Load	14300	11200	26600	21800	13200	10800
	Spandrel	24900	24900			19400	19400
	Weight of Col	1200	1200	1900	1900	1300	1300
	Total	209100	183100	151000	111500	216700	194500
BSMT FLOOR	Floor Load	13700	11700	25400	21800	12600	10800
	Spandrel	26700	26700			24500	24500
	Weight of Col.	1200	1200	1900	1900	1300	1300
	Total	250700	222700	178300	135200	255100	231100
	Material of Column.	2-10"-15# [s 2-9/16" Pls.		2-9"-13.25# [s 2-3/8" Pls.		2-10"-15# [s 2-5/8" Pls.	
FOOTING	Footing	222700		135200		231100	

COLUMN SHEET

		Col. No. 15		Col No. 16		Col No 17.	
		Load on Column	Load on Footing	Load on Column	Load on Footing	Load on Column	Load on Footing
ATTIC FLOOR	Roof Load	25800	15900	22200	11900	22200	11900
	Sky Light Load	1200	700	1200	700		
	Elevator Load	9700	9700	10000	10000		
	Column	200	200	200	200	200	200
	Total	36900	26500	33600	22800	22400	12100
	Material of Column.	2-6"-8# [s Latt.		2-6"-8#-[s Latt.		2 6"-8#-[s Latt.	
4TH+5TH FLOOR	Floor Load	54000	36100	40600	27200	27500	18300
	Spandrel			39000	39000	26100	26100
	Column	400	400	400	400	400	400
	Total	91300	63000	113600	89400	76400	56900
	Material of Column.	2-7"-9.75# [s 2-¼" Pls.		2-7"-9.75# [s 2-¼" Pls.		2-7"-9.75# [s 2-¼" Pls.	
3RD FLOOR	Floor Load	28100	20300	21000	15200	14300	10300
	Spandrel			20000	20000	11900	11900
	Column	500	500	500	500	500	500
	Total.	119900	83800	155100	125100	103100	74600
2ND FLOOR	Floor Load	25900	20300	19400	15200	13100	10300
	Spandrel						
	Column	500	500	500	500	500	500
	Total.	146300	104600	175000	140800	116700	90400
	Material of Column.	2-8"-11.25#-[s 2-5/16" Pls.		2-8"-11.25# [s 2-5/16" Pls.		2-8"-11.25 [s 2-5/16 Pls.	
1ST FLOOR	Floor Load	25800	20300	19400	15200	12500	10300
	Spandrel						
	Column	1000	1000	1000	1000	1000	1000
	Total.	173100	125900	195400	157000	130200	101700
BSMT. FLOOR	Floor Load	23600	20300	17700	15200	12000	10300
	Spandrel						
	Column	1000	1000	1000	1000	1000	1000
	Total	197700	147200	214100	173200	143200	113000
	Material of Column	2-10"-15#-[s 2-3/8" Pls.		2-10"-15# [s 2-3/8" Pls.		2-10"-15# [s 2-3/8" Pls.	
FOOTING	Footing.	14720		173200		113.000	

COLUMN SHEET

		Col. No 13		Col. No. 21.		Col. No. 22	
		Load on Column	Load on Footing	Load on Column.	Load on Footing	Load on Column	Load on Footing
5TH FLOOR	Roof Load	25800	15900	4300	2600	6400	4000
	Spandrel	49000	49000	21200	21200	13700	13700
	Floor Load	11600	8100	6500	4600	9800	6900
	Elevators						
	Wt. of Col.	300	300	300	300	300	300
	Total.	86700	73300	32300	28700	30200	24900
4TH FLOOR	Floor Load	16300	10500	9100	5900	13700	8800
	Spandrel	23000	23000	23000	23000	17000	17000
	Wt. of Col.	300	300	300	300	300	300
	Total.	126300	107100	64700	57900	61200	51000
	Material of Column.	2-8"-11.25# [s 2-¼"Pls.		2-8"-11.25# [s Latt.		2-7"-9.75# [s Latt.	
3RD FLOOR	Floor Load	14500	10500	8200	5900	12300	8800
	Spandrel	24000	24000	32400	32400	20800	20800
	Wt of Col.	600	600	400	400	400	400
	Total.	165400	142200	105700	96600	94700	81000
2ND FLOOR	Floor Load	13400	10500	7600	5900	4600	3600
	Spandrel	19400	19400	17000	17000		
	Wt of Col.	600	600	4000	400	400	400
	Total.	198800	172700	134300	119900	99700	85000
	Material of Column.	2-8"-11.25# [s 2-½"Pls.		2-8"-11.25# [s 2-¼"Pls.		2-7"-9.75# [s 2-¼" Pls.	
1ST FLOOR	Floor Load	12800	10500	7200	5900	4400	3600
	Spandrel	19400	19400	16300	16300		
	Wt. of col.	1400	1400	900	900	700	700
	Total.	232400	204000	158700	143000	104800	89300
BSMT. FLOOR	Floor Load	12200	10500	6900	5900	4200	3600
	Spandrel	24500	24500	22000	22000		
	Wt. of. Col.	1400	1400	900	900	700	700
	Total.	270500	240400	188500	171800	109700	93600
	Material of Column	2-10"-20# [s 2-9/16" Pls.		2-10-15# [s 2-5/16" Pls.		2-8"-1125# [s. 2-¼" Pls.	
FOOTING	Footing.	240400		171800		93600	

COLUMN SHEET

		Col. No. 18		Cols No. 19-20		Cols. No. 23-24	
		Load on Column	Load on Footing	Load on Column	Load on Footing	Load on Column	Load on Footing
ATTIC FLOOR	Roof Load	6500	4000	1300	800		
	Elevator Load	10000	10000	10000	10000	10000	10000
	Column	300	300	200	200	200	200
	Skylight.					1200	700
	Total	16800	14300	11500	11000	11400	10900
	Material of Column	2-6"-8# [s Latt.		2 6"-8# [s Latt.		2-6"-8# [s Latt.	
4TH+5TH FLOOR	Floor Load	23500	16000			10000	3000
	Spandrel	37500	37500	17400	17400	27100	27100
	column	800	800	500	500	500	500
	Total.	78600	68600	29400	28900	49000	41500
	Material of Column	2-7"-9.75# [s 2-¼" P/s.		2-6"-8# [s Latt.		2-6"-8#-[s 2-¼" P/s.	
3RD FLOOR	Floor Load	12500	9000				
	Spandrel	22000	22000	9200	9200		
	Column.	500	500	300	300		
	Total	113600	124300	38900	38400		
2ND FLOOR	Floor load	11500	9000				
	Spandrel	19700	19700	11000	11000		
	Column	500	500	300	300		
	Total	145300	129300	50200	49700		
	Material of Column	2-8"-11.25# [s 2-5/16 P/s		2-6"x 8#-[2 ¼" P/s.			
1ST FLOOR	Floor Load	11000	9000				
	Spandrel	19700	19700	8800	8800		
	Column	1000	1000	400	400		
	Total	177000	159000	59400	58900		
BSMT FLOOR	Floor Load	10500	9000				
	Spandrel	20400	20400	16100	16100		
	Column	1000	1000	400	400		
	Total	208900	189400	75900	75400		
	Material of Column.	2-10"-15# [s 2-3/8" P/s.		2-7"-9.75# [s 2-¼" P/s.			
FOOTING	Footing	189400		75400			

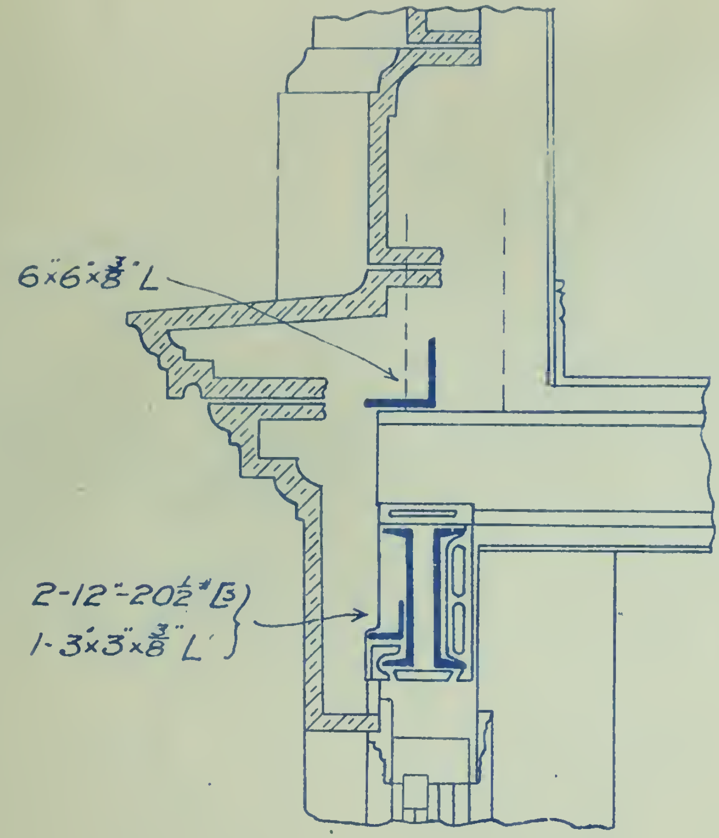


Fig. 9. Spandrel Section Attic Floor

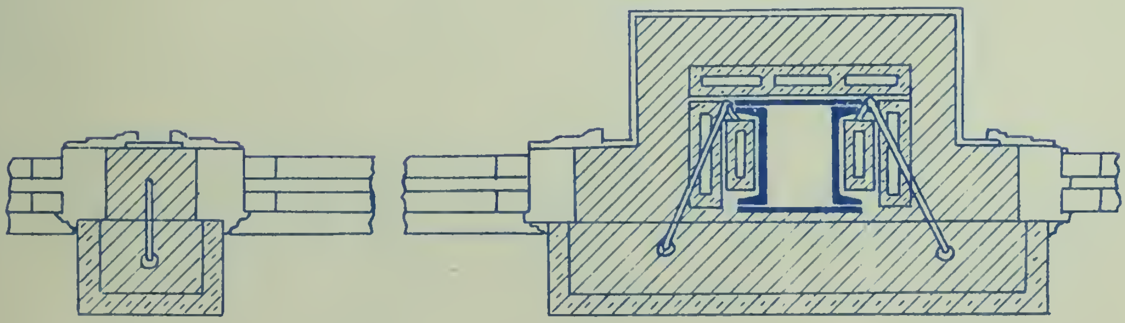


Fig. 11. Section through Pilaster above Fifth Floor

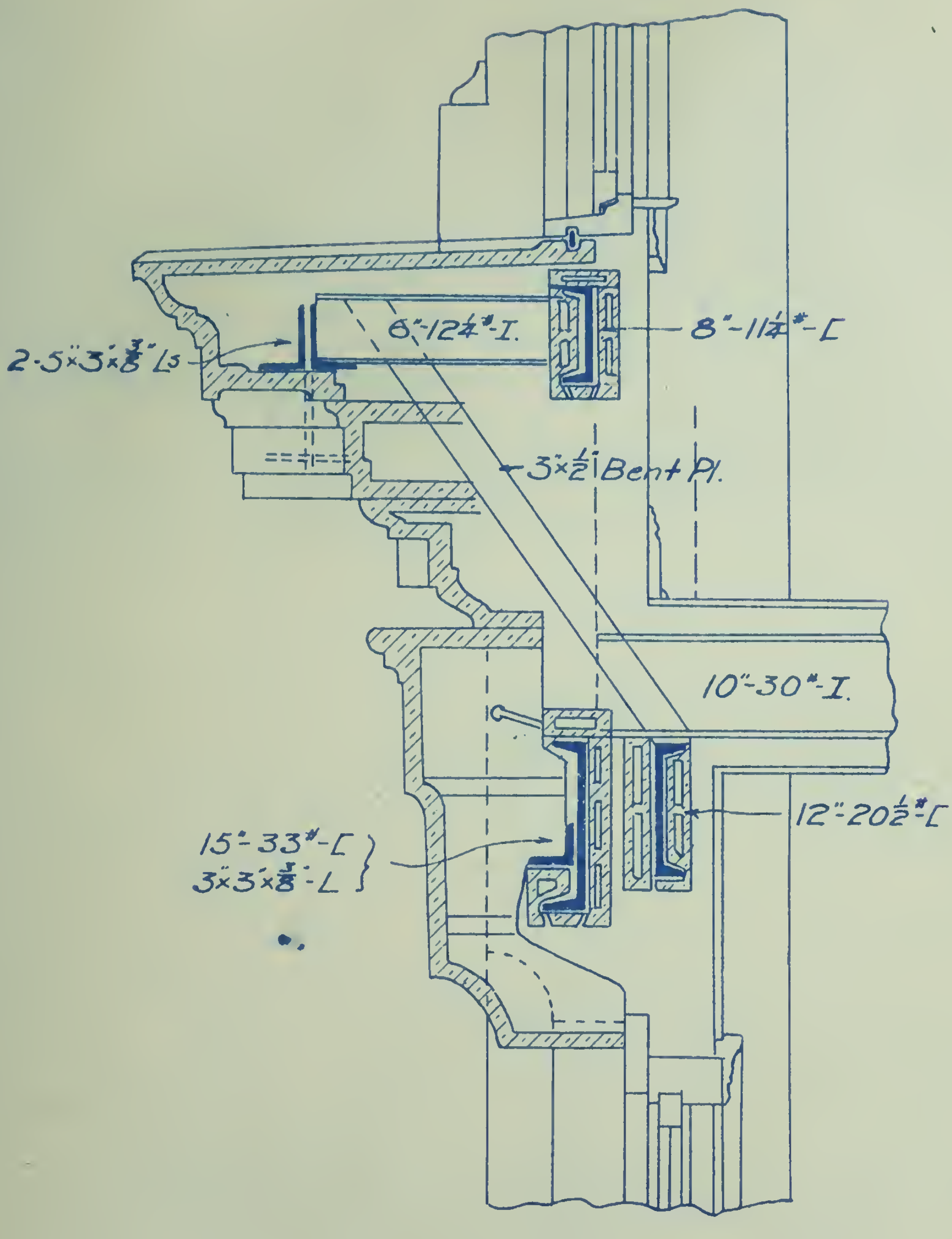
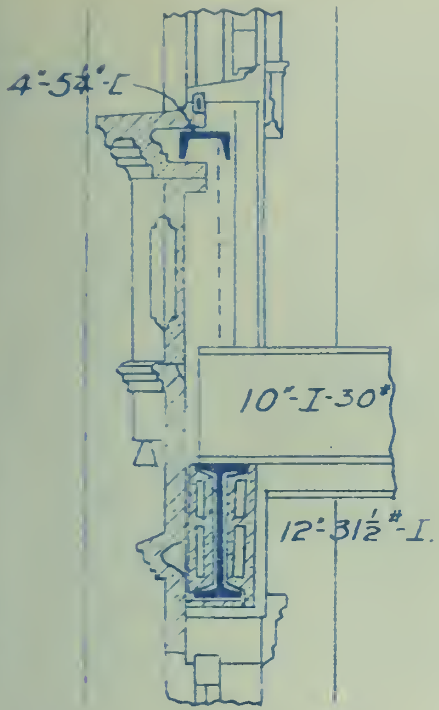


Fig 10 Spandrel Section Fifth Floor.



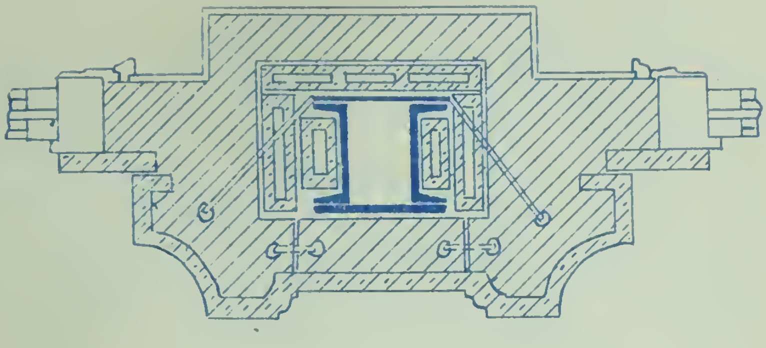


Fig. 13. Section through Pilaster Second Third and Fourth Floors.

Fig. 12. Spandrel Section Second to Fourth Floor.

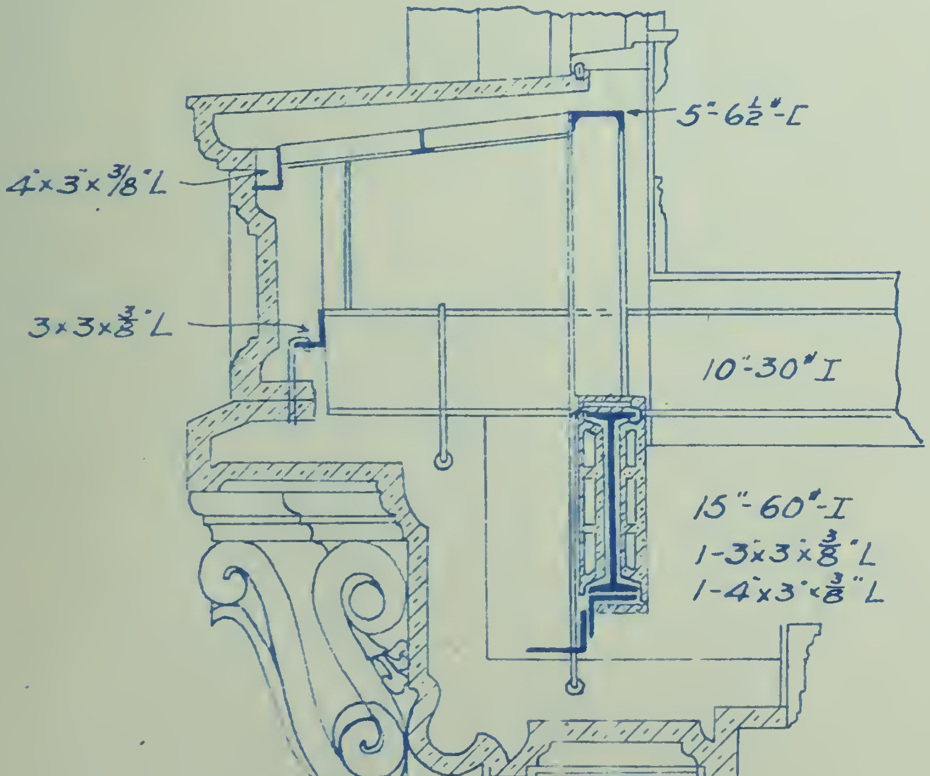


Fig. 14 Section through Entrane Arch Second Floor.

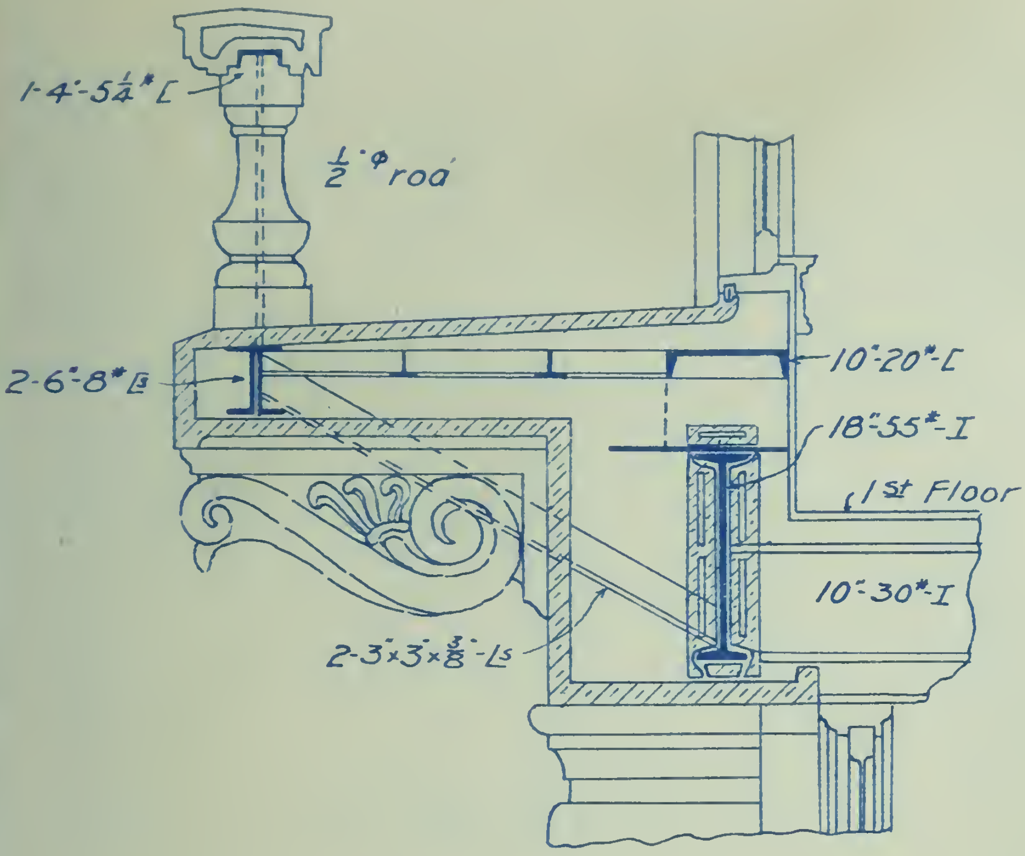


Fig. 15 Spandrel Section First Floor.

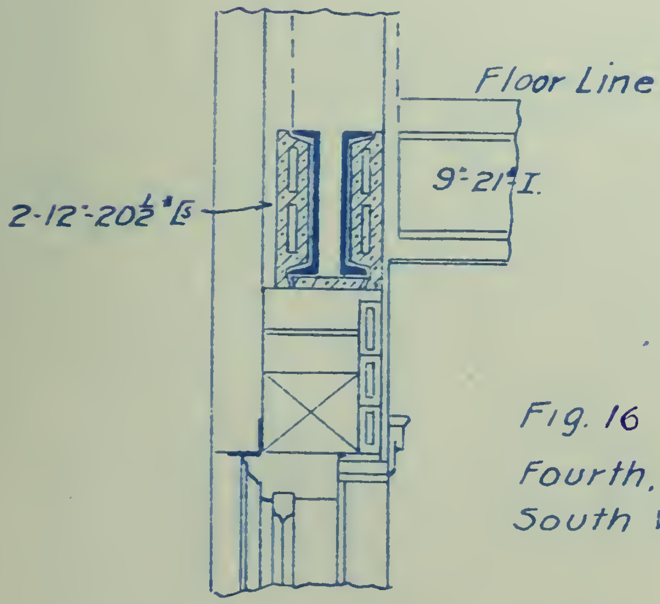
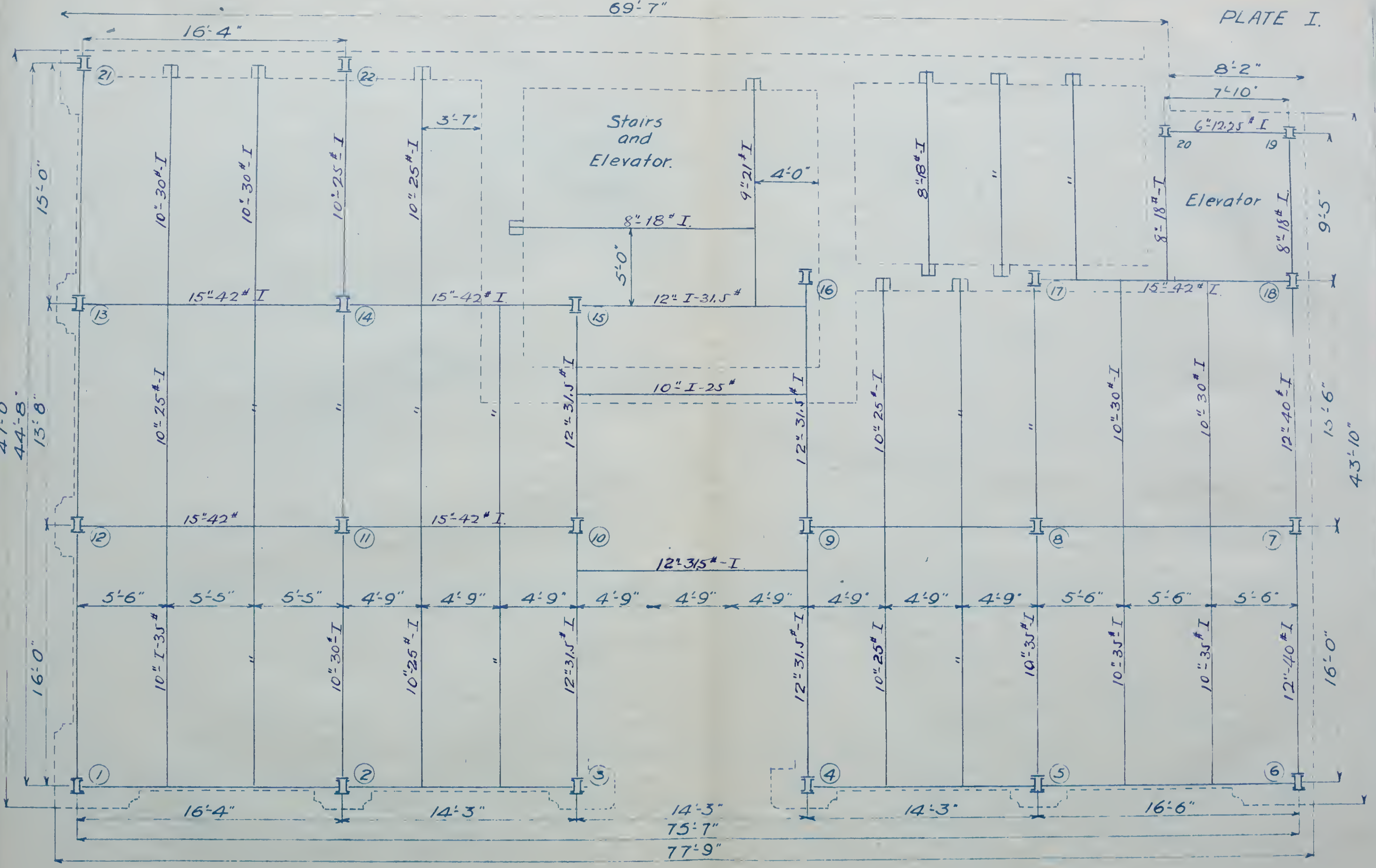


Fig. 16 Spandrel Section
Fourth, Fifth and Attic Floor
South Wall.

PLATE I.

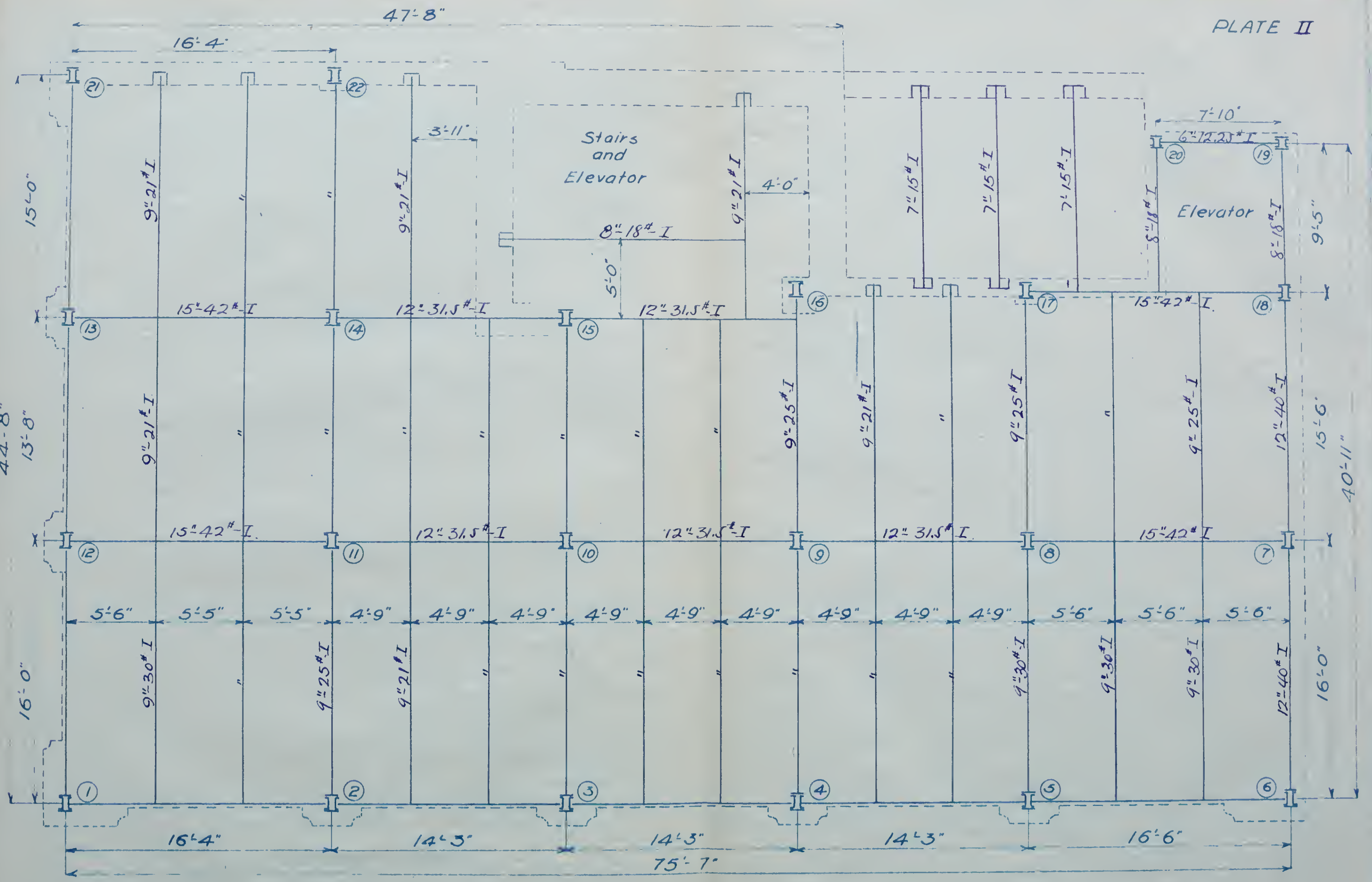
PLATE I.



FIRST FLOOR FRAMING PLAN.

PLATE II.

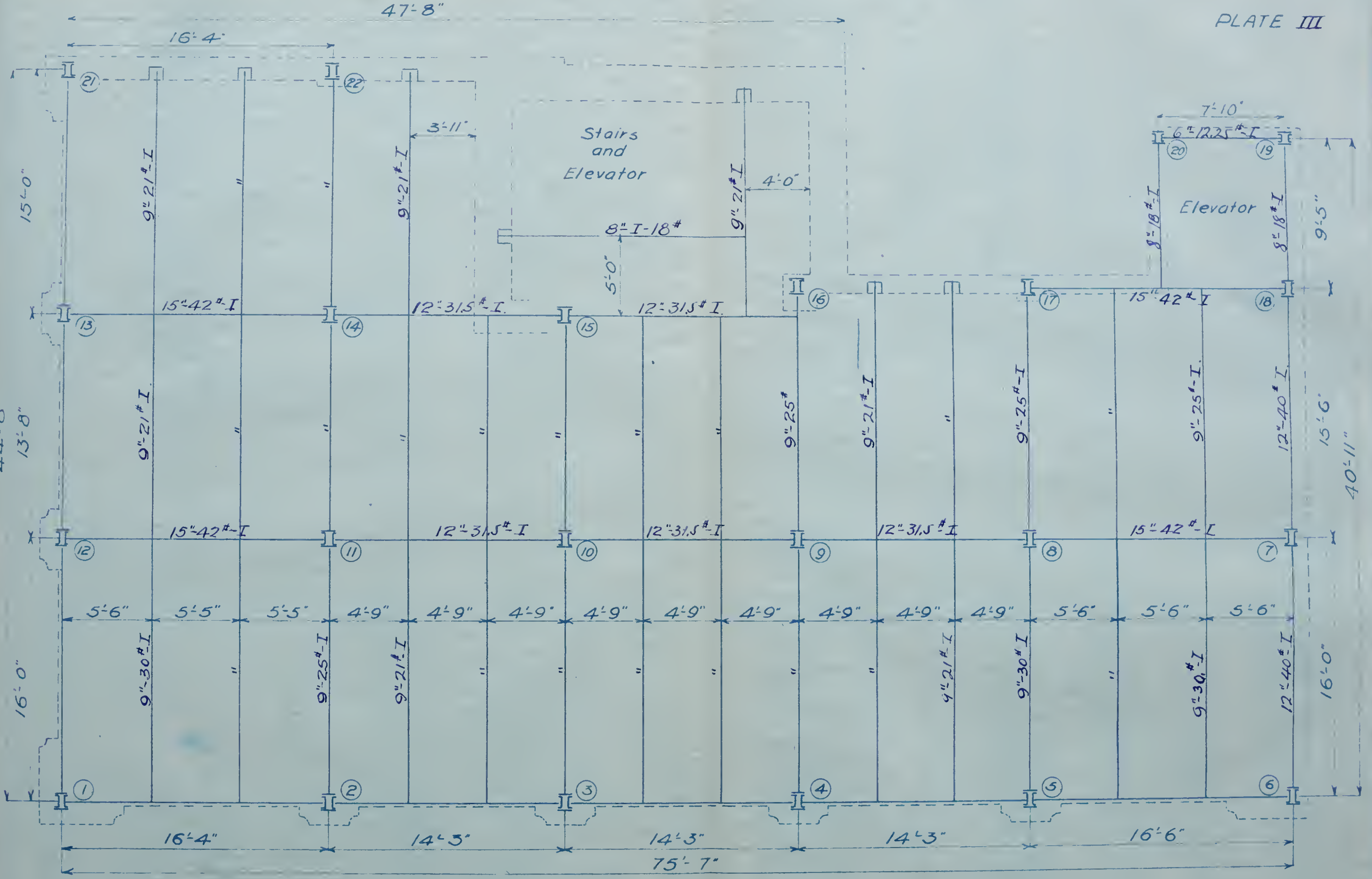
PLATE II



SECOND AND THIRD FLOOR FRAMING PLAN.

PLATE III.

PLATE III



FOURTH FLOOR FRAMING PLAN.

PLATE IV

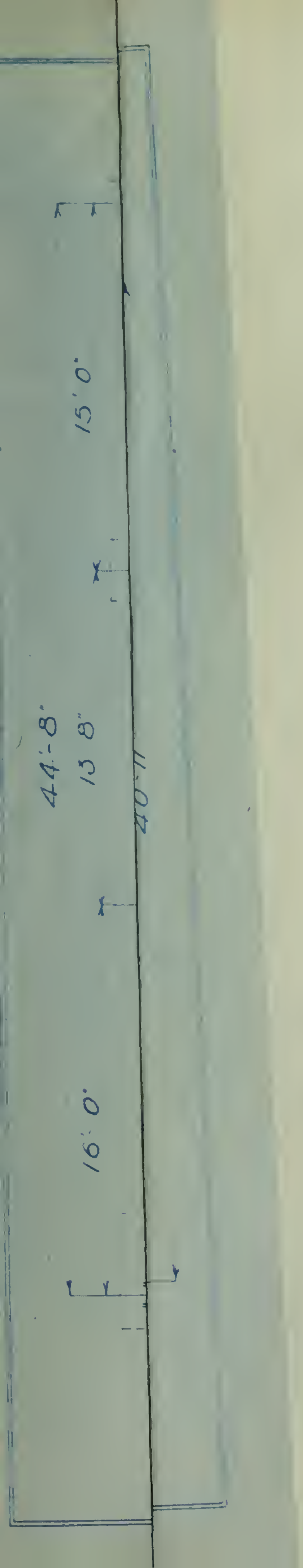
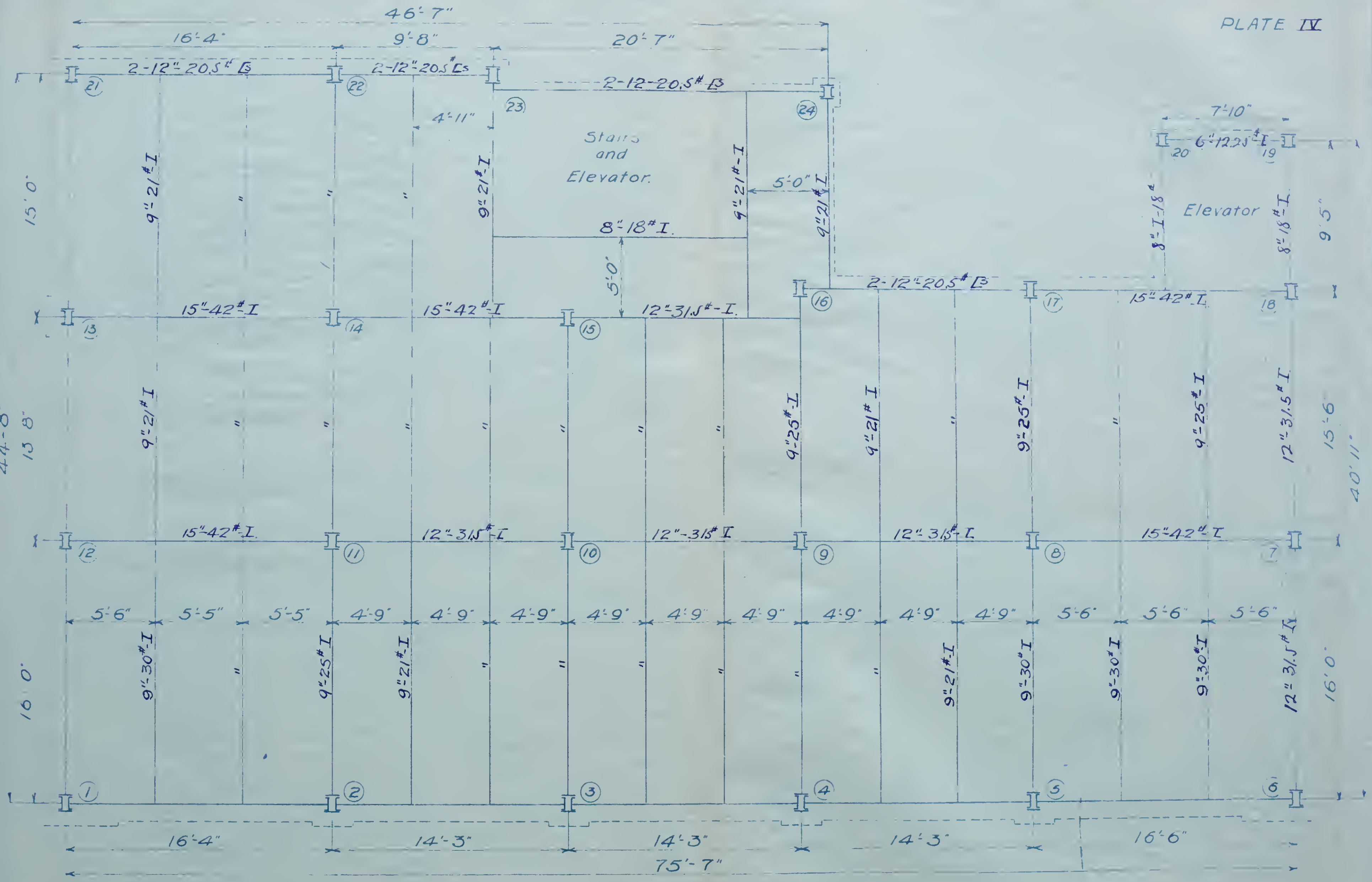


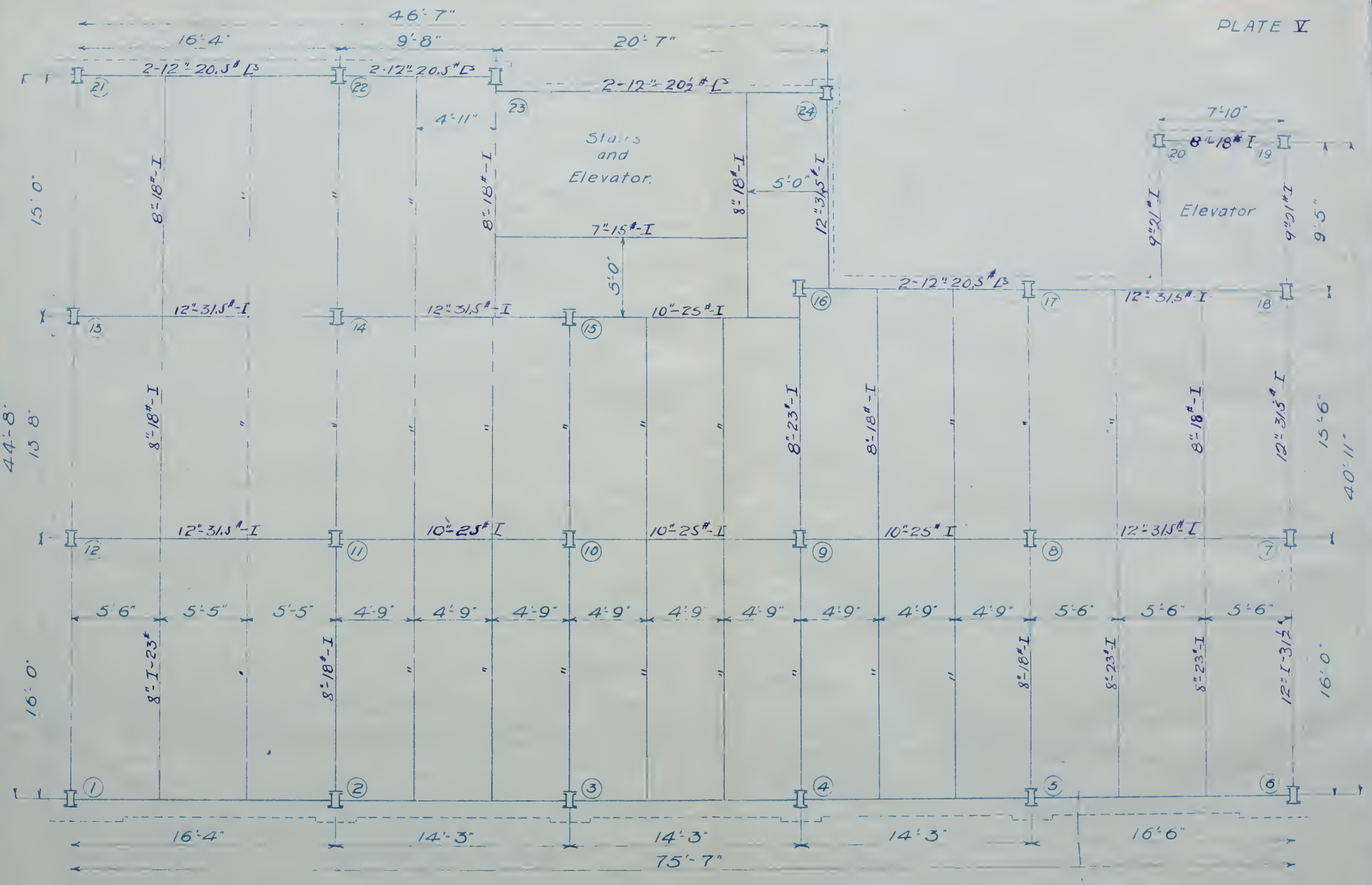
PLATE IV



FIFTH FLOOR FRAMING PLAN

PLATE V.

PLATE V



ATTIC FLOOR FRAMING PLAN

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